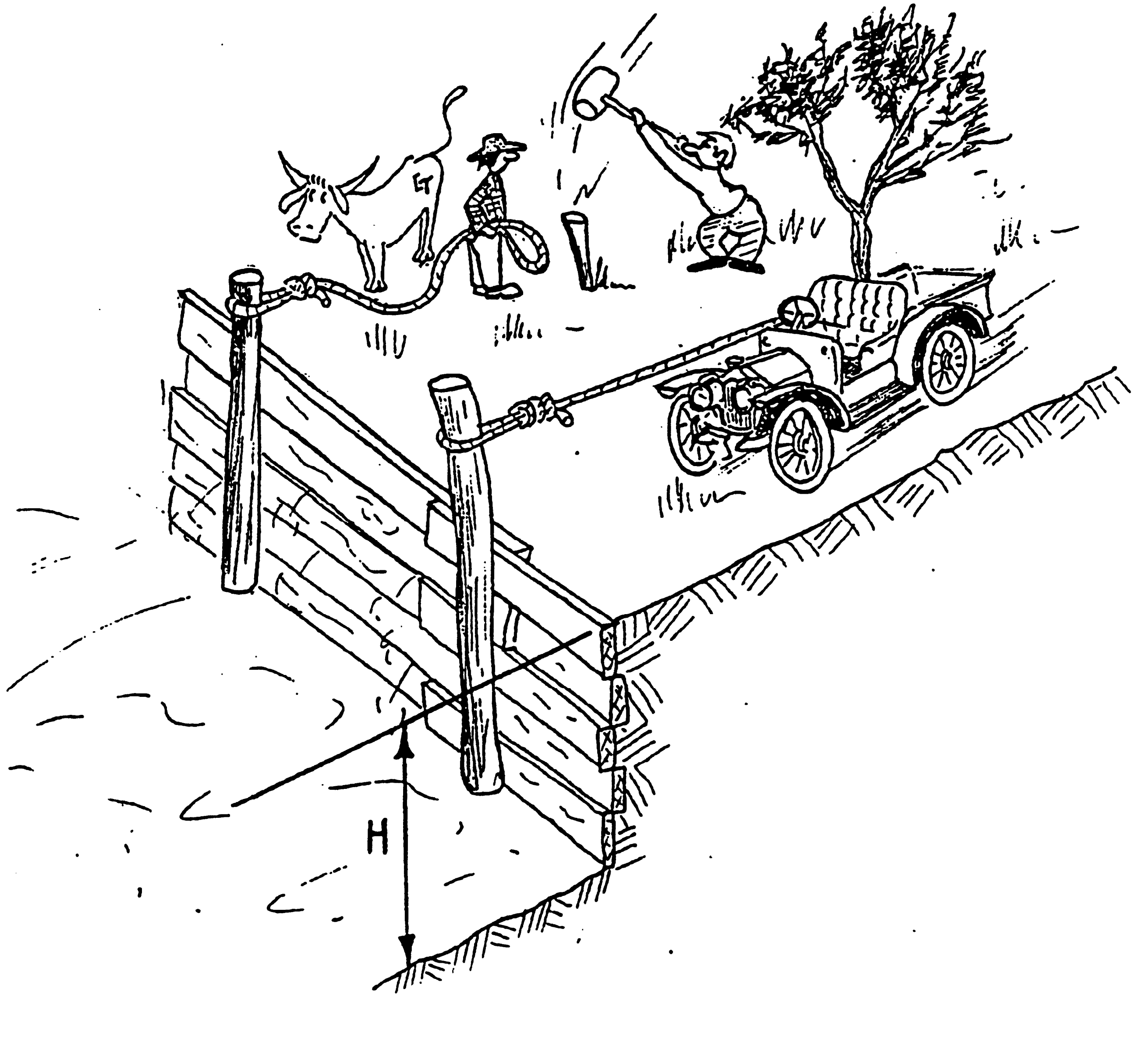
TIEBACK SYSTEMS



ANCHORED SHORING SYSTEMS

Anchored shoring systems used for temporary shoring are primarily two types; stressed anchors (typically tiebacks) composed of high strength steel bars or strands grouted into a drilled hole and passive unstressed anchors (typically tie rods with concrete deadmen or anchor piles). It is common to use nongravity retaining walls to retain the soil with anchors (from one or more tiers) providing additional lateral resistance.

Nongravity cantilevered walls may engage discrete vertical elements with structural facing elements for the retention of soil or may be of a type that uses continuous vertical wall elements that also form the structural facing. Typical discrete vertical elements used for temporary shoring are steel piles with facing elements being timber lagging or steel plates. A common material for continuous vertical wall. elements is steel sheet piling.

As used in this manual, nongravity cantilevered walls with discrete vertical elements will be referred to as soldier pile walls' and those walls with continuous elements will be referred to as sheet pile walls'.

Nongravity cantilevered walls derive lateral resistance through embedment of vertical wall elements and support retained soil with facing elements. The discrete vertical elements typically extend deeper into the ground than the facing to provide vertical and lateral support.

The overall stability of anchored shoring systems and the required strength of its members depends on the interaction of a number of factors, such as the relative stiffness of the members, the depth of piling penetration, the stiffness and strength of the soil, the length of tiebacks, or tierods and the amount of anchor movement. Tiedback systems can be considered flexible systems that allow active pressure to develop; however, if sufficient tieback force is applied and the shoring system is sufficiently rigid, the system may approximate a restrained system.

Shoring systems anchored with passive an&hors will not be covered in this chapter. These types of systems will normally experience more movement than would tiedback systems, and therefore would not be suitable for shoring used to protect. adjacent structures or utilities. The design pressure diagrams, structural analysis and general design considerations detailed in this chapter are applicable to tiedback or strutted shoring systems. The design of deadman anchors may be found in Chapter 11, "Special Conditions". Examples of strutted systems or systems supported by rakers are included in Chapter 8, "Sheet Piling", and in Appendix G, "Sample Problems".

Descriptions of single-tier and multi-tier tiedback shoring. systems along with several sample problems are included in this chapter to demonstrate current technology.

SINGLE-TIER TIEDBACK SYSTEM

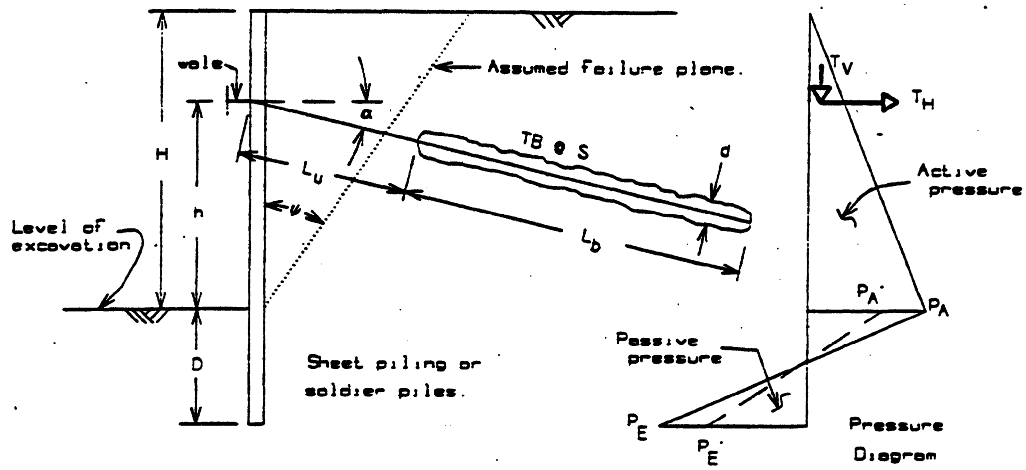


FIGURE 9-1

Nomenclature for Figures 9-1 and 9-2[1]

H = Depth of excavation
D = Embedment depth of piling
h = Height of tieback above level of excavation - generally about 0.75H
T_H = The horizontal component of tieback design force
T_V = The vertical component of the tieback design force
s = Horizontal spacing of tieback
d = Diameter of drill hole for tieback
ψ = Angle between assumed failure plane and vertical
α = Angle of inclination from horizontal of tieback
L_b = Bonded length of tieback
L_u = Unbonded length of tieback

[1]Expanded definition of terms on following pages

SINGLE-TIER TIEDBACK SYSTEM Refer to Figure 9-1

Soil Pressure Values for Cohesionless Soil:

Sheet Pile Shoring:

$P_A = K_a \gamma H \cos\delta$ = Maximum active pressure at the bottom of the excavation.

$P_E = [K_p \gamma D - K_a(D + H)]\cos\delta$ = Maximum passive resisting pressure.

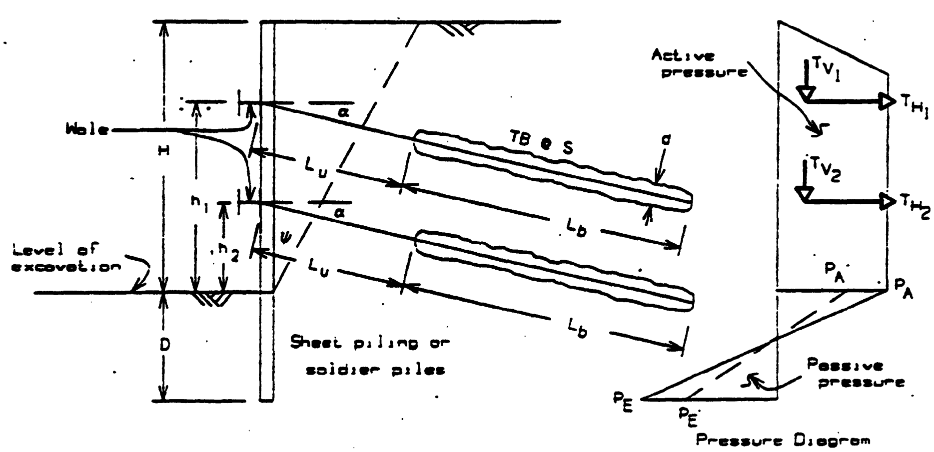
Soldier Pile Shoring:

The active pressure and passive resisting pressure acting on soldier piles below the depth of excavation elevation are reduced by the arching factor (f), defined in Chapter 10, "Soldier Piles," to permit sheet pile type analysis.

$P_A' = fP_A$ and $P_E' = fP_E$

Note: Alternatively a trapezoidal shaped distribution of the design active pressure acting over the height "H" (similar to that used for multi-tier tiedback systems) may be used in lieu of the triangular shaped diagram shown. This method produces a larger tieback force and less embedment depth. The most conservative practice is to check tiebacks using the trapezoidal distribution and embedment depth using the triangular distribution of pressure.

MULTI-TIER TIEDBACK SYSTEM



h_1, h_2 = Vertical location of tieback

FIGURE 9-2

MULTI-TIER TIEDBACK SYSTEM Refer to Figure 9-2

Soil Pressure Values for Cohesionless Soil

Sheet Pile Shoring:

$$P_A = 0.65K_A\gamma H\cos\delta$$

$$P_E = (K_P - K_A)\gamma D\cos\delta - P_A$$

Soldier Pile Shoring:

$$P_A' = fP_A$$

$$P_E' = fP_E$$

General Nomenclature

The embedded portion of the piling below level of excavation. The embedment depth and the horizontal component of the tieback design force required are determined by analyzing the active, passive, and surcharge pressures acting on the piling. A factor of safety is achieved by increasing the calculated embedment depth an additional 20% to 40%. The higher percentage should be used when soil properties are derived from log of test borings or other soil information and not determined from laboratory or in-situ tests used specifically to determine soil strength.

ψ Angle is $\approx 45° - \phi/2$ (for level surface). Values of ψ commonly vary between 20° to 35° depending upon the type of soil. For design of temporary shoring systems, it is normally acceptable to consider the failure plane to start at the elevation of the bottom of the excavation and extend upward at an angle ψ from vertical. For sloping or irregular surfaces, a wedge failure or similar type analysis may be necessary to predict the location of the failure plane (see example in Appendix H, "Memos").

α Angle of inclination of tieback from horizontal. Normally 10° - 15° is used for the angle α to facilitate the placement of grout or concrete. Angles up to 45° may be used to reduce the tieback length, reach stronger soil layers, or to avoid obstacles.

L_b Bonded length of the tieback, which is also referred to as the anchor length of the tieback. The required bonded length depends on the soil or rock properties, the anchor type, and the required anchor capacity.

L_u Unbonded length of tieback. Unbonded length is normally specified to start at some minimum distance past the failure plane to ensure that no portion of the bonded length falls within the failure wedge. Accurate determination of this length depends on how well-known the soil properties are and how accurately the location of the failure plane can be predicted. To ensure that the bonded length falls beyond the failure-plane it is common practice to extend the unbonded length about 5 feet beyond the assumed failure plane. The minimum recommended unbonded length is 15 feet.

CONSTRUCTION SEOUENCE

The construction sequence for an anchored sheet-pile or soldier pile system must be considered when making an engineering analysis. Different loads are imposed on the system before and after the completion of a level of tieback anchors. An analysis should be included for each stage of construction and an analysis may be needed for each stage of anchor removal during backfilling operations.

TIEDBACK ANCHOR SYSTEMS

There are many variations or configurations of tieback anchor systems. The tension element of a tieback may be either prestressing strands or bars using either single or multiple elements. Tiebacks may be anchored against wales, piles, or anchorblocks which are placed directly on the soil. The example problems in this chapter illustrate the use of tiebacks with several different types of shoring systems.

Figure. 9-3 illustrates a typical temporary tieback anchor. In this diagram, a bar tendon system is shown; strand systems are similar.

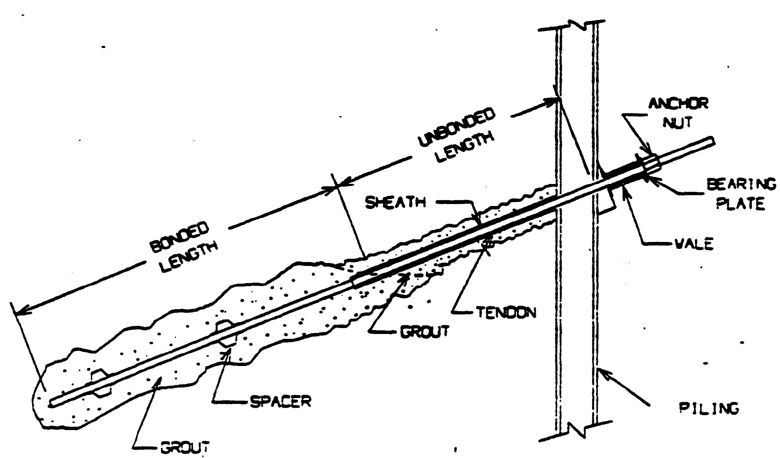


FIGURE 9-3

The more common components,. criteria, and materials used in conjunction with tiedback shoring systems are listed below;

Piling, Sheet piling and soldierpiles. See Chapter 12 "Construction," for common materials and allowable stresses.

Wale - These components transfer the resultant of the earth pressure from the piling to the tieback anchor. A design overstress of 33% is permitted for wales when proof testing the tieback anchor. Anchors for temporary work are often anchored directly against the soldier piling through holes or slots made in the flanges, eliminating the need for wales. Bearing stiffeners and flange cover plates are generally added to the pile section to compensate for the loss of section. A structural analysis of this cut section should always be required.

Tendon - Tieback-tendons are generally the same high strength bars or strands used in prestressing structural concrete.

The anchorage of the tieback tendons at the shoring members consists of bearing plates and anchor nuts for bar tendons and bearing plates, anchor head and strand wedges for strand tendons. The details of the anchorage must accommodate the inclination of the tieback relative to the face of the shoring members. Items that may be used to accomplish this are shims or wedge plates placed between the bearing plate and soldier pile or between the wale and sheet piling or soldier piles. Also for bar tendons spherical anchor nuts with special bearing washers plus wedge washers if needed or specially machined anchor plates may be used.

The tendon should be centered within the drilled hole within its bonded length. This is accomplished by the use of centralizers (spacers) adequately spaced to prevent the tendon from contacting the sides of the drilled hole or by installation with the use of a hollow stem auger.

Stress - Allowable tensile stress values are-based on a percentage of the minimum tensile strength (F_{pu}) of the tendons as indicated below: l

Bars: F_{pu} = 150 to 160 ksi
Strand: F_{pu} = 270 ksi

(Check manufacturers data for actual ultimate strength)

Allowable tensile stresses:

At design load $f_t \leq 0.6\ F_{pu}$
At proof load $f_t \leq 0.8\ F_{pu}$

(Both conditions must be checked)

Grout - A flowable portland cement mixture of grout or concrete which encapsulates the tendon and fills the drilled hole within the bonded length. Generally a neat cement grout is used in drilled holes of diameters up to 8 inches. A sand-cement mixture is used for hole diameters greater than 8 inches. An aggregate concrete mix is comnonly used in very large holes. Type I or II cement is commonly recommended for tiebacks. Type III cement may be used when high early strength is desired. Grout, with very few exceptions, should always be injected at the bottom of the drilled hole. This method ensures complete grouting and will displace any water that has accumulated in the hole.

Tieback anchor

There are several different types of tieback anchors. Their capacity depends on a number of interrelated factors:

Location - amount of overburden above the tieback
Drilling method and drilled hole configuration
Strength and type of the soil
Relative density of the soil
Grouting method
Tendon type, size, and shape

Typical shapes of drilled holes for tieback anchors are depicted in Figure 9-4.

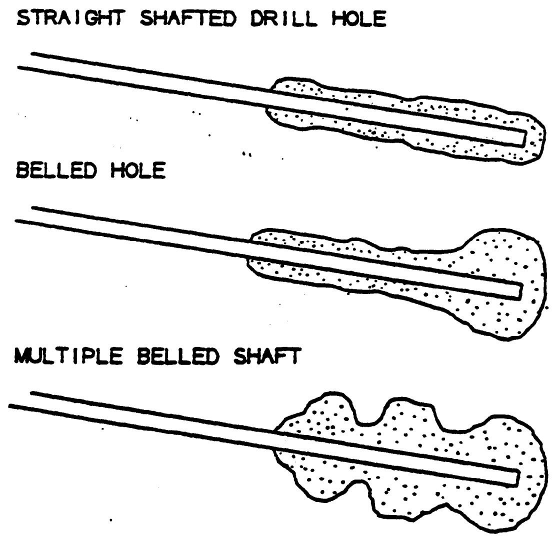


FIGURE 9-4

This is the simplest type and the one encountered most often.

In this case the resistance is a combination of perimeter bond and bearing against the soil.

Similar to above, this type of anchor is also referred to as under-reamed. It is used in stiff cohesive soil. The soil must be stiff enough to prevent collapse of the under-reams or drill hole in the anchor length.

The presence of water either introduced during drilling or existing ground water can cause significant reduction in anchor capacity when using a rotary drilling method in some cohesive soils (generally the softer clays).

High pressure grouting of 150 psi or greater in granular soils can result in significantly greater tieback capacity then by tremie or low pressure grouting methods. High pressure grouting is seldom used for temporary tieback systems.

Regrouting of tieback anchors has been used successfully to increase the capacity of an anchor. This method involves the placing of high pressure grout in a previously formed anchor. Regrouting breaks up the previously placed anchor grout and disperses new grout into the anchor zone; compressing the soil and forming an enlarged bulb of grout thereby increasing the anchor capacity. Regrouting is done through a separate grout tube installed with the anchor tendon. The separate grout tube will generally have sealed ports uniformly spaced along its length which open under pressure allowing the grout to exit into the previously formed anchor.

Due to the many factors involved, the determination of anchor capacity can vary quite widely. Proof tests or performance tests of the tiebacks are needed to confirm the anchor capacity. A Federal publication, the FHWA/RD-82/047 report on tiebacks, provides considerable information for estimating tieback capacities for the various types of tieback anchors. Also see "Supplemental Tieback Information" in Appendix E.

Bond capacity is the resistance to pull out of the tieback which is developed by the interaction of the anchor grout (or concrete) surface with the soil along the bonded length.

Determining or estimating the bond (resisting) capacity is a prime element in the design of a tieback anchor.

Included with some shoring designs there may be a Soils Laboratory report which will contain recommended value for the bond capacity to be used for tieback anchor design. The appropriateness of the value of the bond capacity will only be proven during tieback testing.

For most of the temporary shoring work normally encountered, the tieback anchors will be straight shafted with low pressure grout placement. For these conditions the following criteria can generally be used for estimating the tieback anchor capacity.

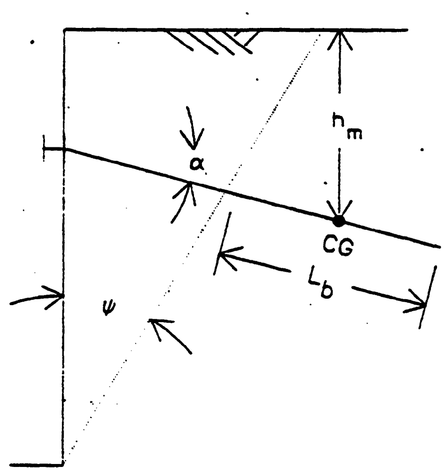


FIGURE 9-5

The FHWA formula for bond is defined as follows:

$$P_{ult} = \pi d L_b \gamma h_m (\tan\phi)$$

Where:

- d = Diameter of drill hole
- L_b = Bonded length of the tieback
- γ = Unit weight of soil
- ϕ = Angle of internal friction of the soil
- CG = Center of L_b = $(L_b/2)$
- h_m = Vertical distance from the ground line to the center of L_b
- ψ = Angle between assumed failure plane and vertical

Forces On The Vertical Members

Tiebacks are generally inclined, therefore the vertical component of the tieback force must be resisted by the vertical member through skin friction on the embedded length of the piling in contact with the soil and by end bearing. Problems with tiedback walls have occurred because of excessive downward wall movement. The pile capacity should always be checked to ensure that it can resist the vertical component of the tieback force. The sheet pile sample problem demonstrates one method to account for the vertical load on the piling.

Ultimate values (without safety factors) for friction and end bearing of piling follow:

For Driven Piling:

Skin Friction

= N/50 tsf for concrete piles

= N/lOO tsf for WF sections

(based on a rectangular perimeter equal to two times the width of the flange added to two times the depth of the section).

End Bearing:

Cohesionless Soil: = 4N tsf

Cohesive Soil:. = $9S_u$ or = 4.5_{qu}

(based on a rectangular perimeter equal to two times the width of the flange added to two times 'the depth of the section).

Special Note: For sheet piling use N/100 for skin friction for depth D on both faces, but do not use end bearing.

For Drilled Piling:

Skin Friction = N/l00 tsf

End Bearing

Cohesionless Soil:= 2N tsf

Cohesive Soil: = $9s_u$ or = 4.5_{qu} (based on the, gross area).

Where N = SPT (Standard Penetration Test) value

Overall (global) System Stability

To ensure overall stability of an an anchored system slope stability analysis may be required in addition. to the general (local) system analysis except when the horizontal component of the anchor is greater than total height of the vertical member. Figure 9-6 depicts the foregoing.

$$a/(H + D) > 1.0$$

Where:

a = The horizontal component of the tieback anchor length

H + D = The vertical member's total length.

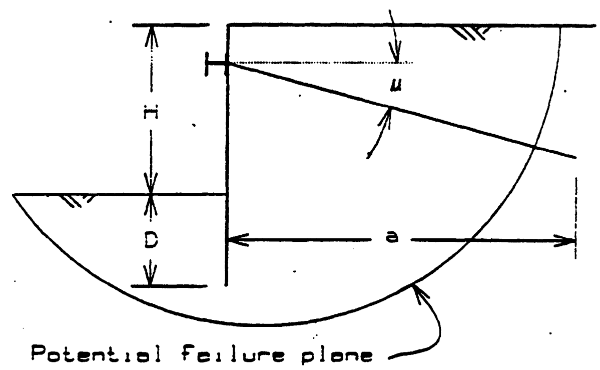


FIGURE 9-6

TESTING TIEBACK ANCHORS

The Contractor is responsible for providing a reasonable test method for verifying the capacity of the tieback anchors after installation. Anchors are tested to assure that they can sustain the design load over time without excessive movement. The need to test anchors is more important when the system will support, or be adjacent to existing structures, and when the system will be in place for an extended period of time.

The number of tiebacks tested; the duration of the test, and the allowable movement, or load loss, specified in the contractor's test methods should take into account the degree of risk to the adjacent surroundings. High risk situations would be cases where settlement or other damage would be experienced by adjacent facilities. See Table 9-1 for a list of minimum recommended criteria for testing temporary tieback anchors.

TIEBACK PROOF TEST CRITERIA

Test Load	Load Hold Duration	Percentage of tiebacks to be load tested
Cohesionless Soils		
Normal Risk		
1.2 to 1.3 Design Load	10 Minutes	10% for each soil type encountered
High Risk		
1.3 Design Load	10 Minutes	20% to 100%
Cohesive Soils		
Normal Risk		
1.2 to 1.3 Design Load	30 Minutes	10%
High Risk		
1.3 Design Load	60 minutes	30% to 100%*

*Use 100% when in soft clay or when ground water is encountered. Use load hold of 60 minutes for 10% and load hold of 10 minutes for remaining 90% of tiebacks.

TABLE 9-1

Generally the shoring plans should include tieback load testing criteria which should minimally consist of proof load test values; frequency of testing (number of anchors to be tested), test load duration, and allowable movement or loss of load permissible during the testing time frame and the anticipated life of the shoring system. The shoring plans should also include the measures that are to be taken when, or if, test anchors fail to meet the specified criteria.

Pressure gages or load cells used for determining test loads should have been recently calibrated by a certified lab, they should be clean and not abused, and they should be in good working order. The calibration dates should be determined and recorded.

Tiebacks which do not satisfy the testing criteria may still have some value. Often an auxiliary tieback may make up for the reduced value of adjacent tiebacks; or additional reduced value tiebacks may be installed to supplement the initial low value tiebacks.

Proof Testing

Proof testing of tiebacks anchors is normally accomplished by applying a sustained proof load to a tieback anchor and measuring anchor movement over a specified period of time. Proof testing may begin after the grout has achieved the desired strength. A specified number of the tieback anchors will be proof tested by the method specified on the Contractor's approved plans (see Table 9-l).

Generally, the unbonded length of a tieback is left ungrouted prior to and during testing (see Figure 9-7). This ensures that only the bonded length is carrying the proof load during testing. It is not desirable to have loads transferred to the soil through grout (or concrete) in the unbonded region since this length is considered to be within the zone of the failure wedge.

As an alternative, for small diameter drilled holes (6 inches or less) a plastic sheathing may be used over the unbonded length of the tendon to separate the tendon from the grout (see Figure 9-3). The sheathing permits the tendon to be grouted full length before proof testing. A void must be left between the top of the grout and the soldier pile to allow for movement of the grout column during testing.

Research has shown that small diameter tiebacks develop most of their capacity in the bonded length despite the additional grout in the unbonded length zone. This phenomenon is not true for larger diameter tieback anchors.

Generally the Contractor will specify an alignment load of 5 to 10% of the design load which is initially applied to the tendon to secure the jack against the anchor head and stabilize the setup. The load is then increased until the proof load is achieved. Generally a maximum amount of time is specified to reach proof load. Once the proof load is attained, the load hold period begins. Movement of the tieback anchor is normally measured by using a dial indicator gage mounted on a tripod independent of the tieback and shoring and positioned in a manner similar to that shown in Figure 9-7.

The tip of the dial indicator gage is positioned against a flat surface perpendicular to the centerline of the tendon (This can be a plate secured to the tendon). The piston of the jack may be used in lieu of a plate if the jack is not going to have to be cycled during the test. As long as the dial indicator gage is mounted independently of the shoring system, only movement of the anchor due to the proof load will be measured. Continuous jacking to maintain the specified proof load during the load hold period is essential to offset losses resulting from anchor creep or movement of the shoring into the supporting soil.

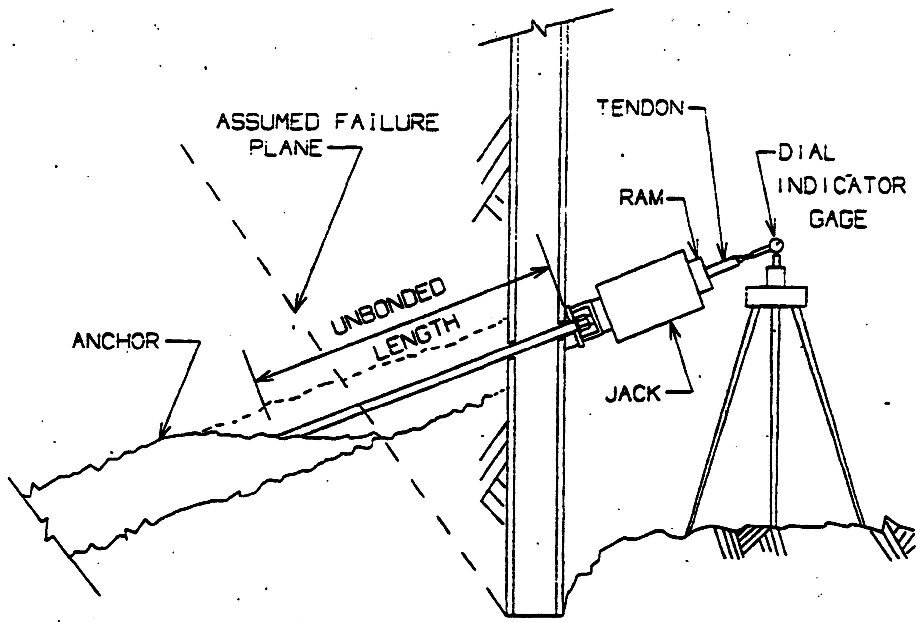


FIGURE 9-7

Measurements from the dial indicator gage are taken periodically during the load hold period. The total movement measured during the load hold period of time is compared to the allowable value indicated on the approved shoring plans to determine the acceptability of the anchor.

It is important that the proof load be reached quickly. When excessive time is taken to reach the proof load, or the proof load is held for an excessive amount of time before beginning. the measurement of creep movement, the creep rate indicated will not be representative. For the proof test to be accurate, the starting time must begin when the proof load is first reached.

As an alternative to measuring movement with a dial indicator gage, the contractor may propose a "lift-off test". A "lift-off test" compares the force on the tieback at seating to the force required to lift the anchor head off of the bearing plate. The comparison should be made over a specified period of time. The lost force can be converted into creep movement to provide an estimate of the amount of creep over-the life of the shoring system.

Use of the "lift-off test" may not accurately predict overall anchor movement. During the time period between lock-off and lift-off, the tieback may creep and the wall may move into the soil. These two components cannot be separated. If the test is done accurately, results are likely to be a conservative measure of anchor movement. The Office of Structure Construction recommends the use of a dial indicator gage to monitor creep rather than lift-off tests.

Evaluation of Creep Movement

Long-term tieback creep can be estimated from measurements taken during initial short term proof testing: In effect, measurements made at the time of proof testing can be extrapolated to determine anticipated total creep over the period the shoring system is in use if it is assumed that the anchor creep is roughly modeled by a curve described by the "log" of time.

The general formula listed below for the determination of the anticipated long term creep is only an estimate of the potential anchor creep and should be used in conjunction with periodic monitoring of the wall movement. This formula will not accurately predict anchor creep for soft cohesive soils.

Based on the assumed creep behavior, the following formula can be utilized to evaluate the long-term effects of creep:

General formula:

$$\Delta_{2\text{-}3} = C[\log_{10}(T_3/T_2)]$$

Where:

$C = \Delta_{1\text{-}2}/[\log_{10}(T_2/T_1)]$

Δ = Creep movement (inches) specified on the plans for times T_1, T_2, or T_3 (or measured in the field)

T_1 = Time of first movement measurement during load hold period (usually 1 minute after proof load is applied)

T_2 = Time of last movement measurement during load hold period

T_3 = Time the shoring system will be in use

If using a 'lift off test' to estimate the creep movement, the following approximation needs to be made for substitution into the above equation:

$$\Delta_{1\text{-}2} \approx (P_1 - P_2)L_u/AE$$

Where:

P_1 = Force at seating $\qquad$ P_2 = Force at lift off

L = L_u + 0 to 5 feet of the bonded length necessary to develop the tendon

A = Area of strand or bar in anchor

E = Modulus of elasticity of the strand or bar in anchor

Sample problem 9-1 demonstrates the calculation of long term creep.

Wall Movement and Settlement

As a rule of thumb, the settlement of the soil behind a tiedback wall, where the tiebacks are locked-off at a high percentage of the design force, can be approximated as equal to the movement at the top of the wall caused by anchor creep and deflection of the piling; Reference is made to the Section titled "Settlement and Deflection" near the end of Chapter 5.

If a shoring system is to be in close proximity to an existing structure where settlement might be-detrimental, significant deflection and creep of the shoring system would not be acceptable: If a shoring system will not affect permanent structures; or when the shoring might support something like a haul road, reasonable lateral movement and settlement can be tolerated.

Performance Testing

Performance testing is similar to, but more extensive, than proof testing. Performance testing is used to establish the movement behavior for a tieback anchor at a particular site. Performance testing is not normally specified for temporary shoring, but it can be utilized to identify the causes of anchor movement. Performance testing consists of incremental loading and unloading of a tieback anchor in conjunction with measuring movement.

Lock-Off Force

The lock-off force is thepercentage of the required design force that the anchor wedges or anchor nut is seated at after seating losses. A value of $0.8T_{DESIGN}$ is typically recommended as the lock-off force but lower or higher values are used to achieve specific design needs.

One method for obtaining the proper lock-off force for strand systems is to insert a shim plate under the anchor head equal to the elastic elongation of the tendon produced by a force equal to the proof load minus the lock-off load. A correction for seating of the wedges in the anchor head is often subtracted from the shim plate thickness. To determine the thickness of the shim plate you may use the following equation:

$$t_{shim} = \frac{(P_{Proof} - P_{lockoff})L}{AE} - \Delta_L$$

where:

t_{shim} = thickness of shim

P_{Proof} = Proof load

$P_{lockoff}$ = Lock–off load

A = Area of tendon steel (bar or strands)

E = Modulus of Elasticity of strand or bar

Δ_L = seating loss

$L \approx$ Elastic length of tendon (usually the unbonded length + 3 to 5 feet of the bonded length necessary to develop the tendon)

Seating loss can vary between 3/8" to 5/8" for strand systems. The seating loss should be determined by the designer of the system and verified during *installation. Often times, wedges are mechanically seated minimizing seating loss resulting in the use of a lesser value for the seating loss. For thread bar systems, seating loss is much less than that for strand systems and can vary between O" to l/16".

After seating the wedges in the anchor head at the proof load, the tendon is loaded, the shim is removed and the whole anchor head assembly is seated against the bearing plate.

CORROSION PROTECTION

The contractor% submittal must address potential corrosion of the tendon after it has been stressed. For very short-term installations in non-corrosive sites corrosion protection may not be necessary. The exposed steel may not be affected by a small amount of corrosion that occurs during its life.

For longer term installations grouting of the bonded and unbonded length-is generally adequate-to minimize corrosion in most non-corrosive sites. Encapsulating *or* coating any ungrouted portions (anchor head, bearing plate, wedges, strand, etc.) of the tieback system may be necessary to guard against corrosion.

For long-term installations or installations in corrosive sites, more elaborate corrosion protection schemes may be necessary (Grease is often used as a corrosion inhibitor). Figure 9-8 depicts tendonsencapsulated in pregreased and pregrouted plastic sheaths generally used for permanent installations.

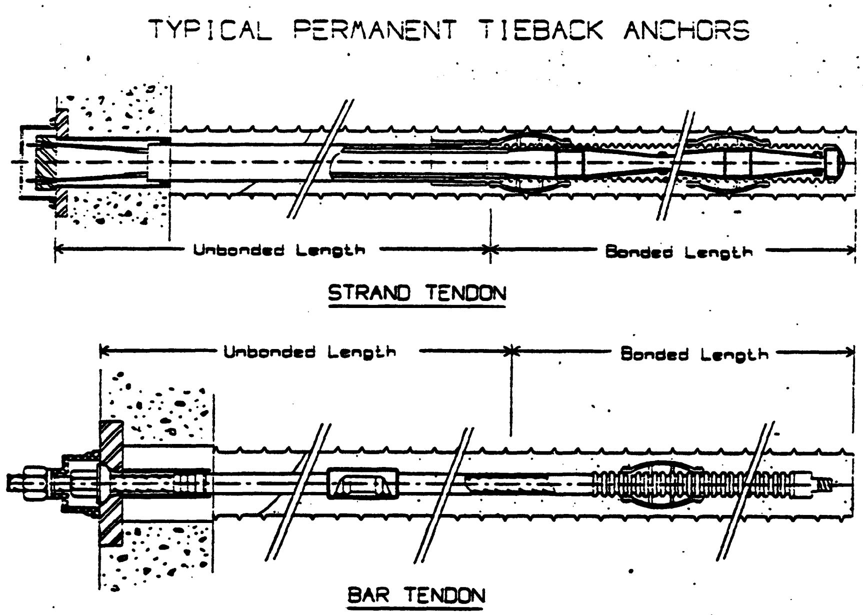


FIGURE 9-8

STEPS FOR CHECKING TIEDBACK SHORING SUBMITTAL

1. Review plan submittal for completeness.
2. Determine K_a and K_P
3. Develop pressure diagrams.
4. Determine forces.
5. Determine the moments around the top of the pile (or some other convenient location).
6. Solve for depth (D), for both lateral and vertical loads, and tieback force (T_H).
7. Check pile section.
8. Check anchor capacity.
9. Check miscellaneous details.
10. Check adequacy of tieback test procedure.
11. Review corrosion proposal,
12. General: Consider effects of wall deflection, and subsequent soil settlement on any surface feature behind the shoring wall.

SAMPLE PROBLEM 9-1 TIEBACK TESTING

Determine the long-term effects of creep.

Measurement and time method:

Given:

The shoring plans indicate that a proof had shall be applied in 2 minutes or less then the load shall be held for ten minutes. The test begins immediately upon reaching the proof load value. Measurements of movement. are to be taken at 1, 4, 6, 8 and 10 minutes. The proof load is to be 133% of the design load. The maximum permissible movement between 1 and 10 minutes of time will not exceed 0.1 inches. All tiebacks are to be tested. The system is anticipated to be in place for 1 year.

Solution:

$A = 0.1$ inches
$T_1 = 1$ minute
$T_2 = 10$ minutes
$T_3 = (1\ Y)\ (365\ D/Y)\ (24\ H/D)\ (60\ M/H) = 525{,}600$ minutes

$$C = \Delta_{1-2}/[\log_{10}(T_2/T_1)] = 0.1/[\log_{10}(10/1)] = 0.1$$

$$\text{Long term } \Delta_{2-3} = (C)\log_{10}(T_3/T_2) = (0.1)\log_{10}(525{,}600/10)$$
$$= 0.47 \text{ inches} = 1/2 \text{ inch}$$

The proof load, and duration of test are reasonable and exceed the minimums shown in Table 9-l. Applying the proof load in. a short period of time and beginning the test immediately upon reaching that load ensure the test results will be meaningful and can be compared to the calculated long term creep movement for the anchor.

If the shoring system was in close proximity to an existing structure that could not tolerate a l/2 inch of settlement the design would not be acceptable. If the shoring would not affect permanent structures or when the shoring might support something like a haul road, the anticipated movement would be tolerable.

Lift off load method:

Given:

Lift off test will be performed 24 hours after wedges are seated (1 minute). The force at seating the wedges will be 83,000 pounds and the lift off force will be no less than 67,900 pounds.

$L \approx 20$ ft which is the unbonded length of 15′ + 5′
$A = 0.647$ in^2
$E = 28\text{x}10^6$ psi
$T_2 = 1$ minute, this is the time the wedges are seated.

$\Delta_{1\text{-}2} \approx ((P_1 - P_2)L)/AE$
$\approx ((83{,}000 - 67{,}900)(20)(12))/((0.647)(28\text{x}10^6))$
≈ 0.2 in

$C \approx 0.2/[\log_{10}(1440/1)]$
≈ 0.06

Long term $\Delta_{2\text{-}3} \approx (C)\log_{10}(T_3/T_2) = (0.06)\log_{10}(525{,}600/1)$
≈ 0.34 inches $\approx 5/16$ inch

SAMPLE PROBLEM 9-2 SINGLE-TIER TIEBACK SHORING WALL

This example problem illustrates the analysis for a single tier tieback sheet pile wall next to a haul road and demonstrates the 'following principles:

- The use of Teng's "Free Earth Support Method" of sheet pile analysis with Rowe's "Moment Reduction Theory" to determine the required depth of embedment (D), the required sheet pile section modulus ($S_{REQUIRED}$), and the design tieback force(T).

- Low pressure grouted anchor tieback analysis.

- Review of proof loading and lock-off loading.

The Contractor% shoring submittal outlined below using PSX32 steel sheet pile *is* to be reviewed for adequacy.

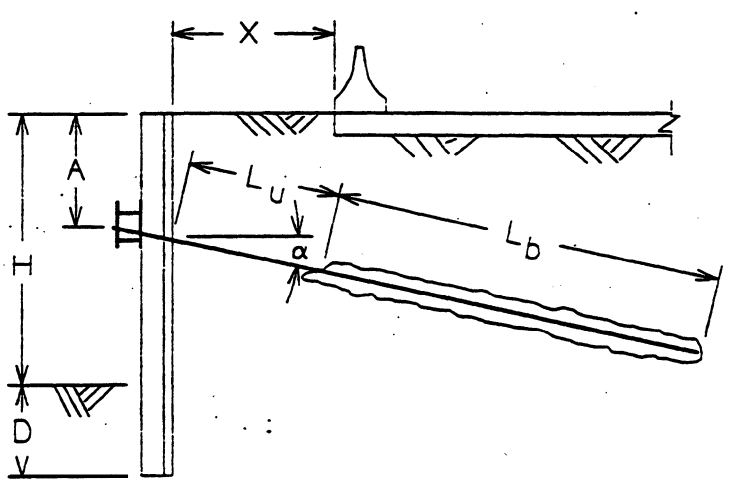


FIGURE 9-9

Soil Properties:

γ = 115 pcf

ϕ = 35°

Dimensions:

H = 15 feet　　X = 10 feet　　A = 3' -6"

L_U= 15 feet　　L_b= 25 feet　　D = 6'-6"

Tieback angle α = 15°　　Tieback spacing = 8'-O"

Anchor Details:

5/8" Dywidag bars at 8' 0" center spacing centered in 6" diameter (d) drilled holes which are to be grouted with low pressure grout.

T_{DESIGN} = 25 Kips
T_{proof} = $(1.3)T_{DESIGN}$

Proof Testing of Tiebacks: (Notes on the shoring plans)

Alternate anchors will be proof tested to T_{PROOF} after the anchor grout has obtained adequate strength.

The exposed end of the anchor rod shall not show movement of more that 2 inches while jacking up to the proof load value.

The proof load (T_{PROOF}) shall be attained and held for 15 minutes. Anchor movement shall not exceed 0.1 inches between 1 and 15 minute, Readings shall be taken at 1, 5, 10 and 15 minutes. The system will be in place approximately 6 months.

Anchors failing the -test criteria shall be replaced.

Analysis:

Top failure wedge width = 15' $\tan(45° - \phi/2)$ = 7.8' < 10'+ 2'.

Since light haul road traffic is to be beyond the active failure wedge limits, the use of minimal friction on the sheet piling for the active condition may be permitted.

For simplified analysis, use the alternate loading of 100 psf for traffic surcharge.

Pressure Diagram:

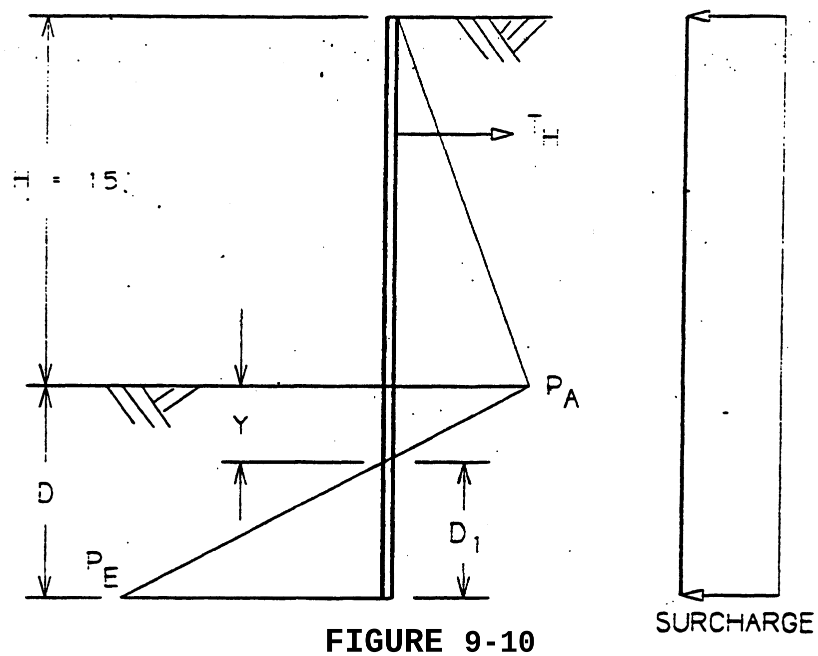


FIGURE 9-10

Use friction angle $\delta = \phi/2 = 17.5°$

From Log-Spiral: $K_a = 0.27$

From Log-Spiral: $K_p = 10.5(0.362) = 3.8$
(It is recommended that friction not be used for the passive condition; especially under conditions of dynamic loading).

$P_A = (\gamma)(H)(K_a)\cos\delta = (115)(15)(0.27)\cos 17.5° = 444.0$ psf

$P_E = \gamma(K_p - K_a)D_1 = 115(3.53)D_1 = 406D_1$ psf

$Y = P_A / \gamma(K_p - K_a) = 444/406 = 1.09$ feet

Horizontal Forces:

$P_1 = P_A(H)/2 = 444(15)/2 = 3{,}330$ Lb/LF

$P_2 = P_A(Y)/2 = 444(1.09)/2 = 242$

$P_{SC} = 100(H + Y + D_1) = 100(16.09 + D_1) = 1{,}609 + 100D_1$

$P_3 = P_E(D_1)/2 = 406D_1(D_1)/2 = 203(D_1)^2$

T_H = Unknown

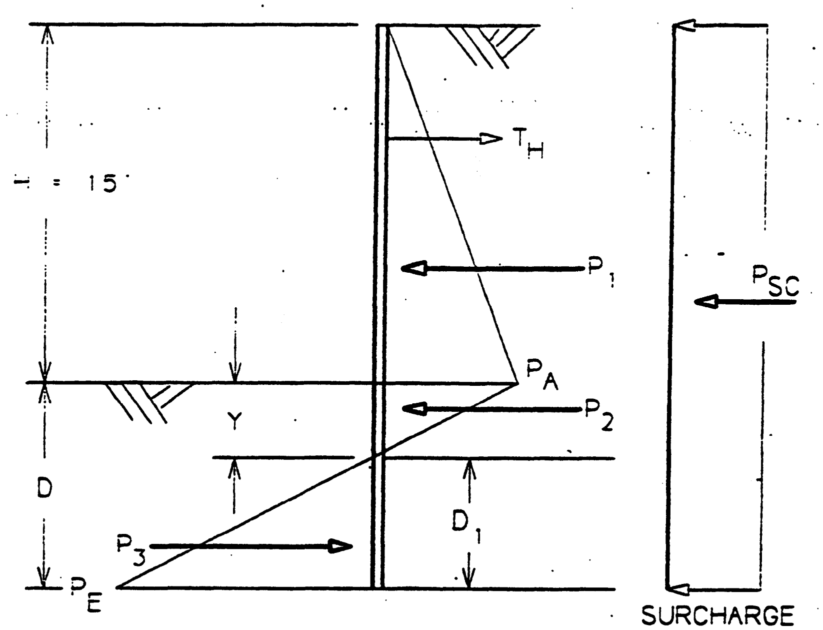


FIGURE 9-11

Sum Forces ($\Sigma F_H = 0$) → + :

$$T_H + P_3 - P_2 - P_1 - P_{SC} = 0$$

$$T_H + 203(D_1)^2 - 242 - 3{,}330 - (1{,}609 + 100D_1) = 0$$

EQUATION 1: $T_H = -203(D_1)^2 + 100D_1 + 5{,}181$

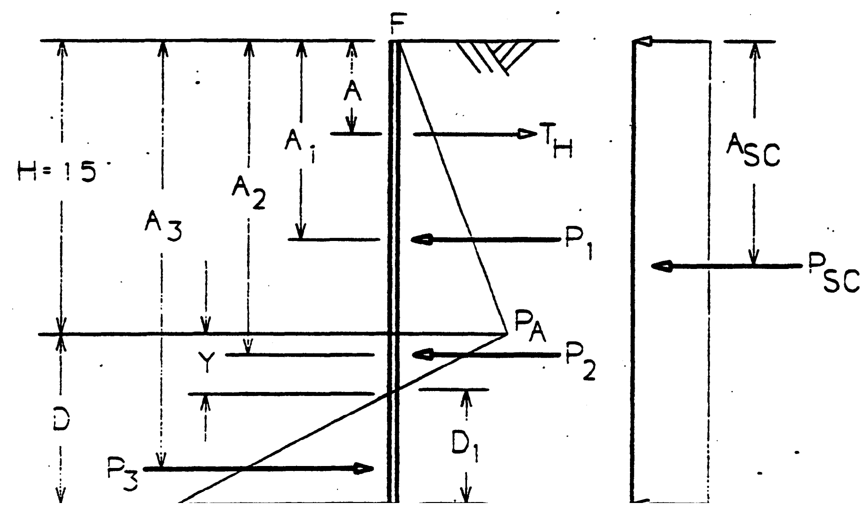


FIGURE 9-12

Moments About Top of Shoring:

$P_1[2/3(H)]$		$3,330[10]$
	$=$	$33,300$
$P_2[15 + Y/3]$	$=$	$242[15.36]$
	$=$	$3,717$
$P_{SC}[(H + D)/2]$	$=$	$P_{SC}[(H+Y+D_1)/2]$
	$=$	$P_{SC}[(16.09+D_1)/2]$
	$=$	$(1,609 + 100D_1)[(16.09 + D_1)]/2$
	$=$	$50(D_1)^2 + 1,609D_1 + 12,944$
$P_3[H + Y + 2/3D_1]$	$=$	$203(D_1)^2[16.09 + 2/3D_1]$
	$=$	$3,266(D_1)^2 + 135(D_1)^3$
$T_H[3.5]$	$=$	$3.5T_H$

Sum Moments ($\sum M_F$) = 0: (Use clockwise moments negative)

$$3.5T_H + 135(D_1)^3 + 3,266(D_1)^2 - 50(D_1)^2 - 1,609D_1 - 12,944$$
$$-3,717 \quad - 33,300 = 0$$

$$3.5T_H = -135(D_1)^3 - 3,216(D_1)^2 + 1,609D_1 + 49,961$$

EQUATION 2:

$$T_H = \quad -38.6(D_1)^3 - 918.9(D_1)^2 + 459.7D_1 + 14,2174.6$$

Equate EQUATION 1 = EQUATION 2 and solve for D:

$$- 38.6(D_1)^3 - 715.9(D_1)^2 + 359.7D_1 + 9,093.6 = 0$$

$$(D_1)^3 + 18.5(D_1)^2 - 9.3D_1 - 235.6 = 0$$

From which $D_1 \approx 3.50'$

then $D \approx 3.50 + 1.09 = 4.6'$

Increase D for factor of safety: $1.4(4.6) = 6.4 \quad < 6'-6"$

Solve EQUATION 1 for T_H:

$$T_H = -203(3.50)^2 + 100(3.50) + 5,181 = 3,044.2 \text{ Lb/LF}$$

With ties at 8'-0" spacing design $T_H = 8(3,044) = 24,352$

Compute Tieback Forces:

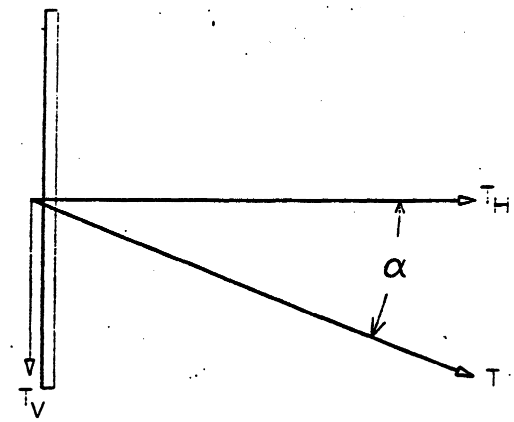


FIGURE 9-13

$T = T_H/\cos\alpha = 24{,}352/\cos 15° = 25{,}211$ Lb > 25K per plans

This difference is too small to be considered significant.

USE T = 25K

$T_H = 25{,}000 \cos 15° = 24{,}148$ Lb $= 24{,}148/8 = 3{,}019$ Lb/LF

$T_V = 25{,}000 \sin 15° = 6{,}470$ Lb

Check Downward Force due to Prestressing:

Resistance to downward force is furnished by the skin. friction on both sides of the embedded sheet piling.

From TABLE 12 N = 27 for $\phi = 35°$

Friction resistance = N/100 tsf = 27/100 = 0.27 tsf

Resistance for 8'-0"
= 0.27(2,000 Lb/ton)(2 sides)(D)(Spacing)
= 0.27(2,000)(2)(6.5)(8)
= 56,160 Lb

Use safety factor of 2:

Resistance = 56,160/2 = 28,080 > T_V = 6,470 Lb OK

Locate Plane of Zero Shear for Sheet Piling:

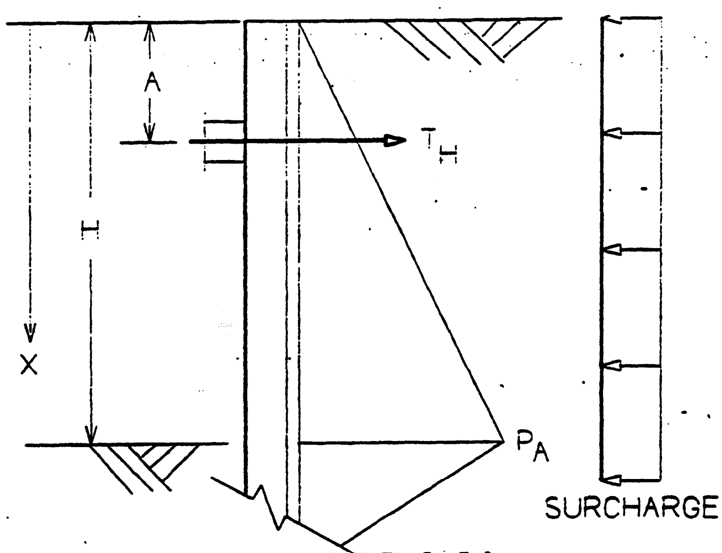


FIGURE 9-14

Assume 3.5' ≤ X ≤ 15'

$K_A\gamma(X^2/2)(\cos\delta) + 100X - T_H = 0$

$[0.27(115)(X^2/2)](\cos 17.5°) + 100X - 3,019 = 0$

$14.8X^2 + 100X - 3,019 = 0$

$X^2 + 6.8X - 204.0 = 0$

The plane of zero shear is where X = 11.3'

Sheet Pile Moment and Section Modulus:

$M_{MAX} = K_A\gamma(X^2)/2(\cos\delta)[X/3] + 100X[X/2] - T_H[X-3.5]$

$= 0.27(115)[(11.3)^2/2](0.95)[11.3/3] + 100(11.3)[11.3/2] - 3,019[7.8]$

$= 7,094 + 6,385 - 23,548 = -10,069$ Ft-Lb/LF

$S_{REQUIRED} = M/f = 10,069(12)/25,000 = 4.83 \text{ in}^3$

For the PSX32 sheet pile section to be used $S = 2.4 \text{ in}^3/\text{LF}$ and $I = 3.7 \text{ in}^4/\text{LF}$ (see Table 19). Sheet pile sections this flexible are not generally used adjacent to traffic or other critical surcharge loads, but are being used here for illustrative purposes.

Since analysis is based on the Free Earth Support Method, Rowe's Moment Reduction Theory may be utilized.

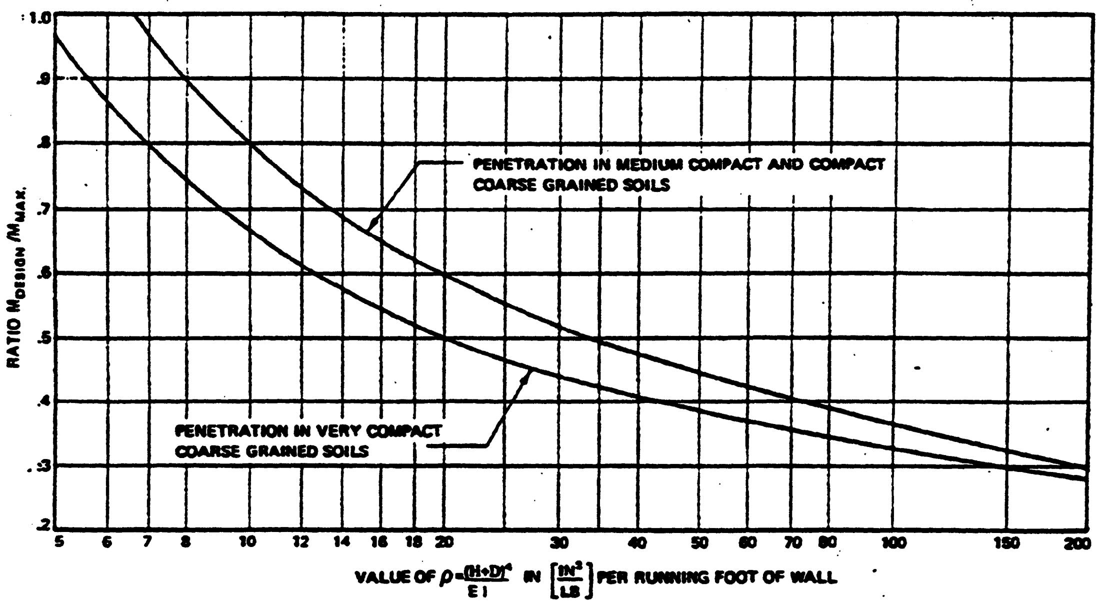


FIGURE 9-15

$\rho = (H + D)^4/EI = [(15 + 6.5)(12)]^4/(29x10^6)(3.7) = 41$

Use the curves in the preceding diagram for, 'PENETRATION IN MEDIUM COMPACT TO COMPACT COARSE GRAINED SOILS', to obtain the moment ratios:

Ratio $M_{DESIGN}/M_{MAX} = 0.47$

$M_{DESIGN} = 0.47(10,069) = 4,732$ Ft-Lb/LF

$f = M/S = 4,732(12)/2.4 = 23,660 < 25,000$ psi OK

<u>Check Anchor Tendon Capacity:</u>

Plan calls for 5/8" Dywidag bars spaced at 8'-0" centers:

$F_{ult} = 157$ ksi

$A_{bar} = 0.28$ in^2

<u>Allowable Bar Capacity:</u>

$T_{DESIGN} \leq 0.6F_{ult}A_{bar} = 0.6(157)(0.28) = 26.4$ K

$T_{PROOF} \leq 0.8F_{ult}A_{bar} = 0.8(157)(0.28) = 35.4$ K

Actual load on bars:

T_{DESIGN} = 25 K < 26.4 K OK
T_{PROOF} = 1.3(25) = 32.5 K < 35.4 K OK

H = 15'
A = 3.5'
L_u = 15'
L_b = 25'
ϕ = 35°
γ = 115 pcf

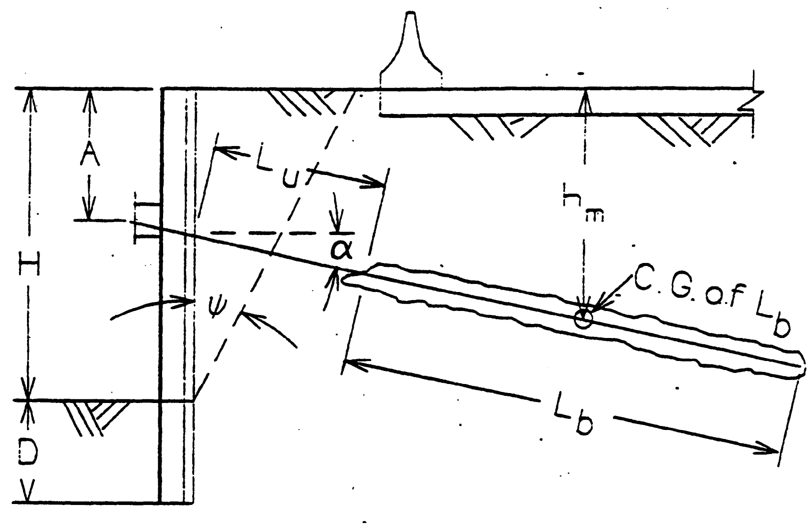


FIGURE 9-16

Assumed
failure wedge angle ψ = 45° - ϕ/2 = 27.5° α = 15°

Check L_u minimum:

L_u minimum = (H - 3.5')(sin ψ)/sin [180° - (90° - α) - ψ)

= (15' - 3.5')sin 27.5°/sin 77.5°

= 5.4' < 15' OK

Determine h_m:

h_m = 3.5 + (L_u + L_b/2)sin α

= 3.5 + (15' + L_b/2)sin 15°

= 7.4' + 0.13L_b

Determine L_b using the FHWA formula:

$$P_{ult} = \pi(d)(L_b)(\gamma)(h_m)\tan\phi$$

$$= \pi(0.5)(L_b)(115)(7.4 + 0.13L_b)\tan 35°$$

$$= 936L_b + 16.4(L_b)^2 = 33{,}650 \text{ Lb}$$

P_{ult} = 33,650 Lb > T_{PROOF} = 32,500 Lb OK

Proof testing will verify actual anchor capacities.

Simplified Wall Stability Check For Single Tier Tieback:

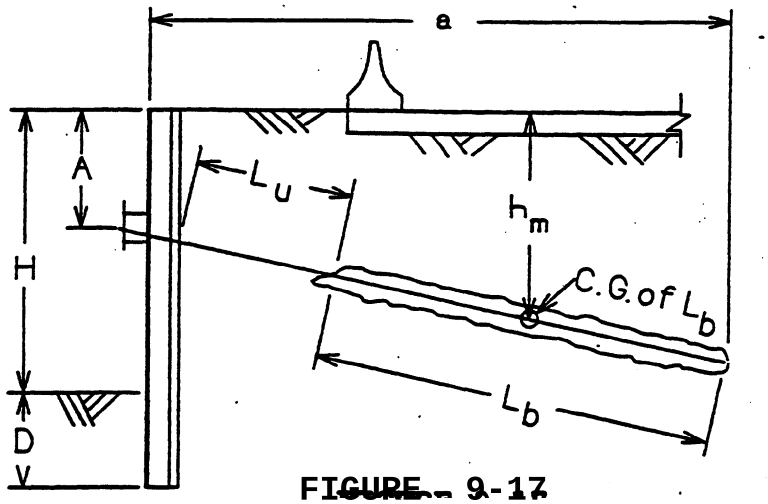


FIGURE 9-17

If a/(H + D) > 1, The wall may be considered stable.

where: a = horizontal component of the tie.

$= (L_u + L_b)\cos\alpha$

$= (15' + 25')\cos 15° = 38.6$ feet

a/(H + D) = 38.6'/(15' + 6.5') = 1.8 > 1 OK

Lock-Off Force:

A value of $0.8T_{DESIGN}$ is typically recommended as a minimum value for low to normal risk conditions. The use of $0.8T_{DESIGN}$ would be satisfactory for this case provided small settlements behind the wall will not be detrimental.

Check Proof Loading:

$T_{PROOF} = 1.3T_{DESIGN} = 1.3(25{,}000) = 32{,}500$ Lb

Δ = 0.1 inch T_1 = 1 minute T_2 = 15 minutes

T_3 = (1/2 Yr.)(365/2 D/Y)(24 H/D)(60 M/H) = 262,800 minutes

$$C = \Delta_{1\text{-}2}/[\log_{10}(T_2/T_1)] = 0.1/[\log_{10}(15/1)] = 0.085$$

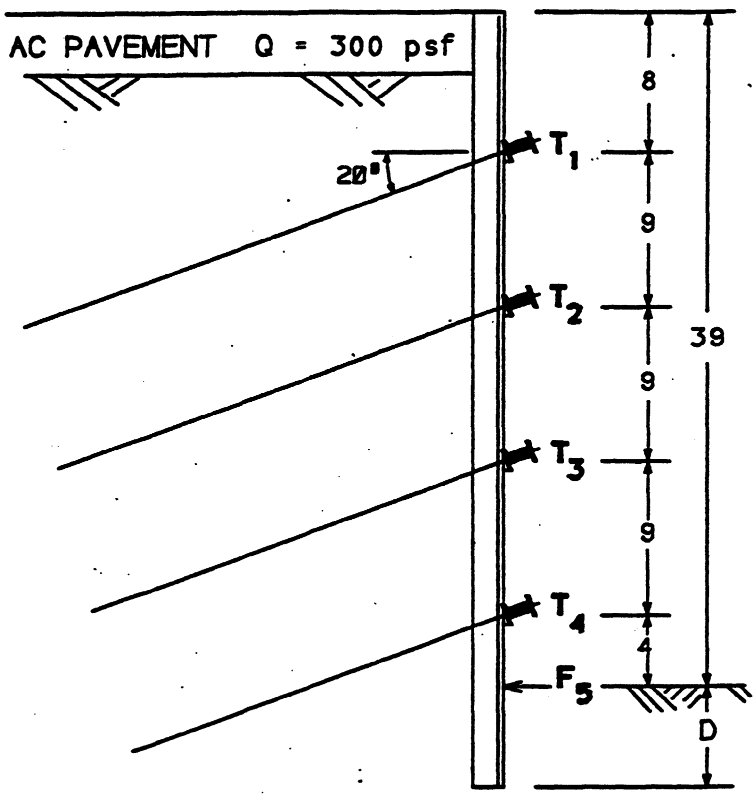
$$\begin{aligned} \text{Long term } \Delta &= (C)\log_{10}(T_3/T_2) \\ &= (0.085)\log_{10}(262{,}800/15) \\ &= 0.36 \text{ in.} \end{aligned}$$

A long term movement of the wall can be approximated but if neither wall movement nor settlement behind the wall will. be detrimental then 0.36 inch would be acceptable.

SAMPLE PROBLEM 9-3 MULTIPLE-TIER TIEBACKS (PART 1)

This is a two part sample problem. The first part is a sample design using simplified criteria assuming the vertical member to be hinged at the depth of excavation. The second part is an analysis of the design and tiebacks using OSC criteria.

PART1



From Soils Report

γ = 115 pcf

ϕ = 35°

C = 0

Kw = 25

K_p = 2.6

FIGURE 9-18

Driven New Steel Sheet Piling: Casteel CS60:

$S = 6.98\ in^3/Ft^2$ of wall

$I = 20.6\ in^4/LF$

$F_b = 25$ ksi

Tiebacks spaced at 7' -6" along with W16 x26 wales

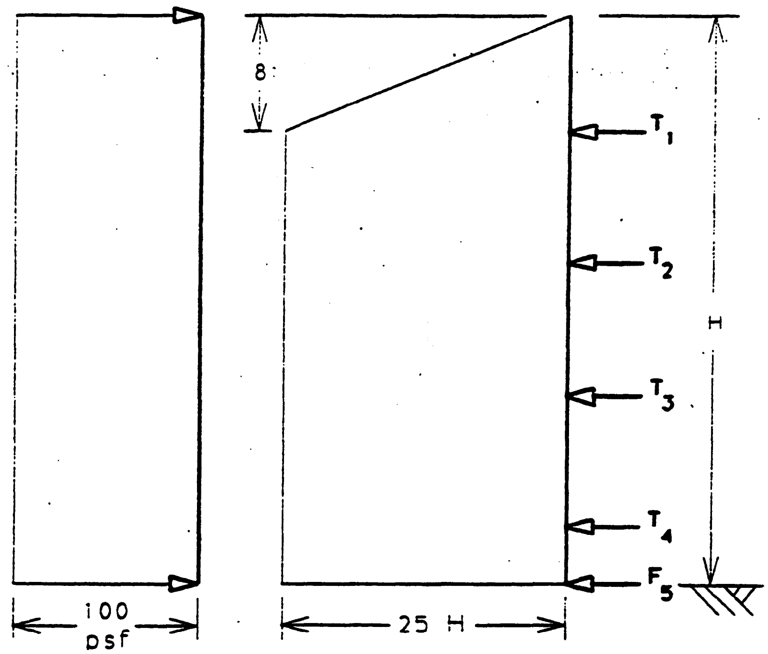


FIGURE 9-19

Shear and moment at T_1 due to cantilever:

V_c = 0.5(0.025)(H)(8) + 0.100(8) = 4.7 Kip/LF

M_c = 3.9(8/3) + 0.8(8/2) = 13.6 K-Ft/LF

COMPUTER PROGRAM RESULTS:

at	Moment (Ft-Kips/LF)	Reaction (Kips/LF)
T_1	13.6	10.4
T_2	5.5	8.5
T_3	8.1	10.3
T_4	5.4	8.0
F_5	0	0.8

Approximate Maximum Positive Span Moments:

span 1 = 1.7 Ft-Kips/LF

2 = 4.1

3 = 4.1

4 = 0.3

Design Moment = Maximum Moment = 13.6 Ft-Kips/LF

Section Modulus Required 13.6(12)/25 = 6.52 < 6.98 in^3 OK

Tieback Forces:

T_1 = 7.5(10.4) = 78 K

T_1 Load = 78/cos 20° = 83 K

T_1 Test = 83/0.8 = 104 K

T_2 = 7.5(8.5) = 63.75 K

T_2 Load = 63.75/cos 20° = 68 K

T_2 Test = 86/0.8 = 108 K

T_3 = 7.5(10.3) = 77.25 K

T_3 Load = 77.25/cos 20° = 82 K

T_3 Test = 82/0.8 = 103 K

T_4 = 7.5(8) = 60 K

T_4 Load = 60/cos 20° = 64 K

T_4 Test = 64/0.8 = 80 K

Depth of Embedment:

Downward Force = (83 + 68 + 82 + 64)sin 20° = 101.58 K

Friction on sheet piling = (78 + 64 + 77 + 60)(0.4) = 111.6

111.6 > 101.58 K

Use passive resistance = 2.6 times the unit weight of the soil.

Passive resistance = 2.6(115) = 300 pcf

The primary portion of the downward force from the tieback load may be acting on one sheet pile, so prorate the load to the pile width of 27.6 inches (2.3 feet):

F_5(7.5/2.3) = 800(7.5/2.3) = 2,609 Lb/Sheetpile

This force needs to be resisted by a passive triangular load having a resistance of 300D for depth D so that:

2,609 = 1/2(300)(2.3)D^2 from which D = 2.75'

Use safety factor of 30%. Design D = 2.75(1.30) = 3.58'
Use D = 4.0 feet

Check Excavation Levels at 2'-0" Below Ties:

EXCAVATE 10'

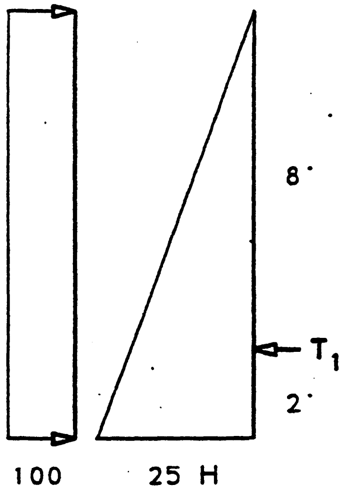


FIGURE 9-20

Before stressing T_1:

$$M_{10'} = 25H(H/2)(H/3) + 100(H)(H/2)$$
$$= 25H^3/6 + 100H^2/2$$
$$= 9{,}167 \text{ Ft-Lb/LF}$$

After stressing T_1:

$$M_{10'} = 9{,}167 - 10{,}400(2)$$
$$= -11{,}633 \text{ Ft-Lb/LF}$$

$11{,}633 < 13{,}600$ Ft-Lb/LF OK

EXCAVATE 19'

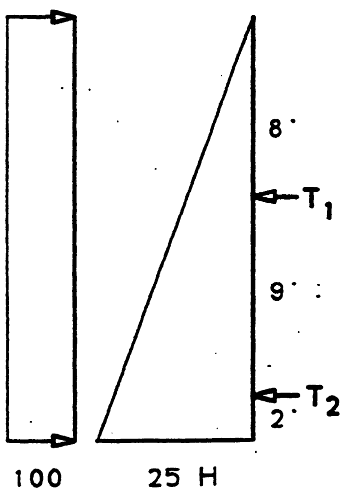


FIGURE 9-21

Before stressing T_2:

$$M_{19'} = 25(19)^3/6 + 100(19)^2/2 - 10{,}400\ (11)$$
$$= -67{,}770 \text{ Ft-Lb/LF} > 13.6 \text{ K-Ft/LF}$$

But which will be resisted the passive soil behind the piling ($K_p > K_a$).

And remainder OK by similar analysis.

MULTIPLE-TIER TIEBACKS (PART 2)

The traveled way pavement width of 15' - 2" ± starts 6 feet from the face of excavation. Surcharge loading for traffic is 300 psf acting vertically.

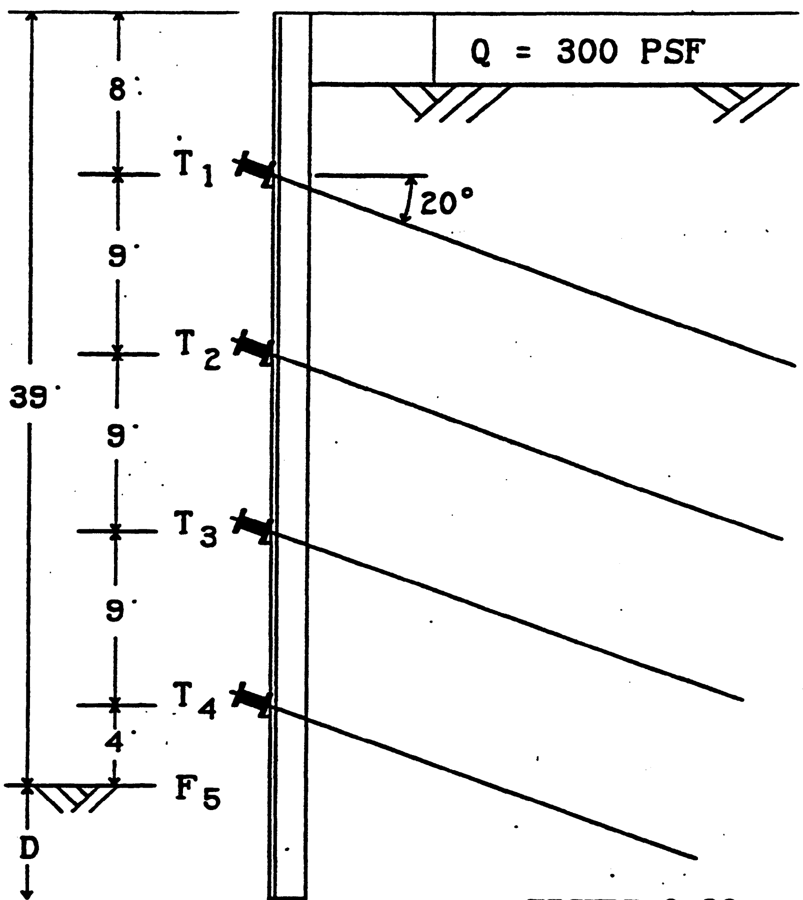


FIGURE 9-22

From Soils Report:

Soil unit weight γ = 115 pcf Wall friction δ = 11°
Internal friction angle ϕ = 35° Cohesion C = 0
Equivalent unit weight Kw = 25 pcf

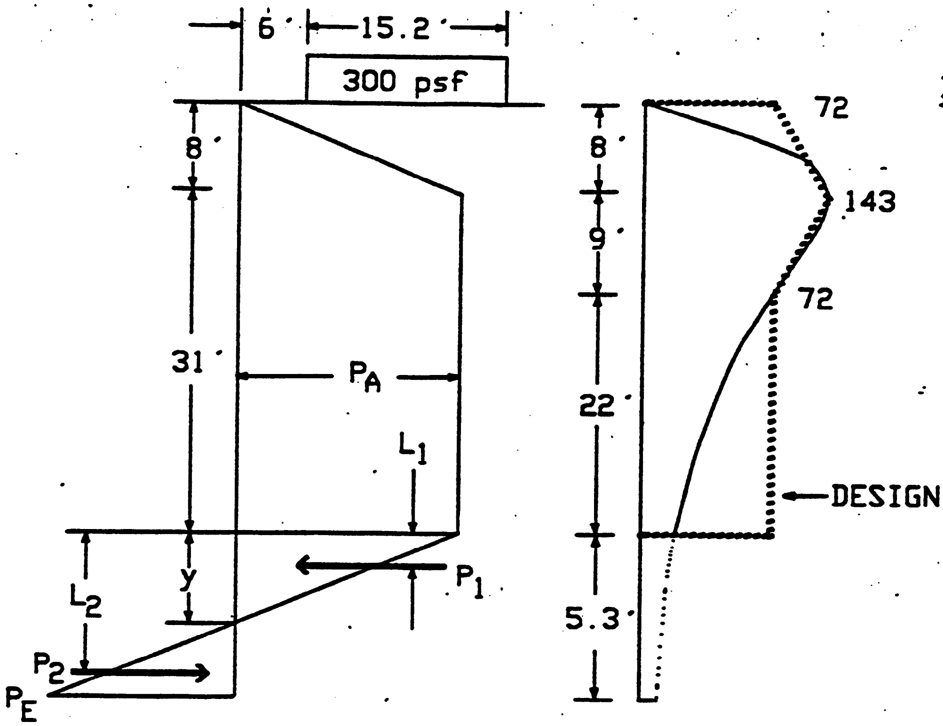
Shoring is driven new steel sheet piling:

Casteel CS60 for which: S = 6.98 in^3/Ft of wall
I = 20.6 in^4/LF
F_b = 25 ksi

Tiebacks spaced at 7' - 6" along W16 x 26 wales

ANCHORED SHORING SYSTEMS

Use the Boussinesq loading for the surcharge and assume this loading-carries to the bottom of the excavation.



DEPTH	PRESSURE
0	0 (72)
2	83
4	130
6	143
8	137
10	123
12	107
14	92
16	79
18	67(72)
44	10(0)

SOIL PRESSURE SURCHARGE PRESSURE

9-23

Use: $K_a = 0.28$, $K_p = (8.4)(0.593) = 4.98$

$P_A = 0.71 K_a \gamma H(\cos \delta) = (0.71)(0.28)(115)(39)(\cos 11°) = 875$ psf

$P_E = \gamma D(K_p - K_a) - P_A = (115)(5.3)(4.70) - 875 = 1{,}990$ psf

$y = P_A/\gamma(K_p - K_a) = 875/(115)(4.70) = 1.62'$

$P_1 = (875)(1.62)/2 = 709$ Lb

$P_2 = (1{,}990)(5.3 - 1.62)/2 = 3{,}662$ Lb

$L_1 = y/3 = 1.62/3 = 0.54'$

$L_2 = y + (D - y)(2/3) = 1.62 + (5.3 - 1.62)(2/3) = 4.07'$

Two approaches may be considered for force P_2:*

1. Consider leg T_4-P_2 as a cantilever. <---Use option #1 for this analysis
2. Consider P_2 an auxiliary support.

* More recent thinking permits a hinge at the excavation line.

Moment Distribution Factors & Fixed End Moments:

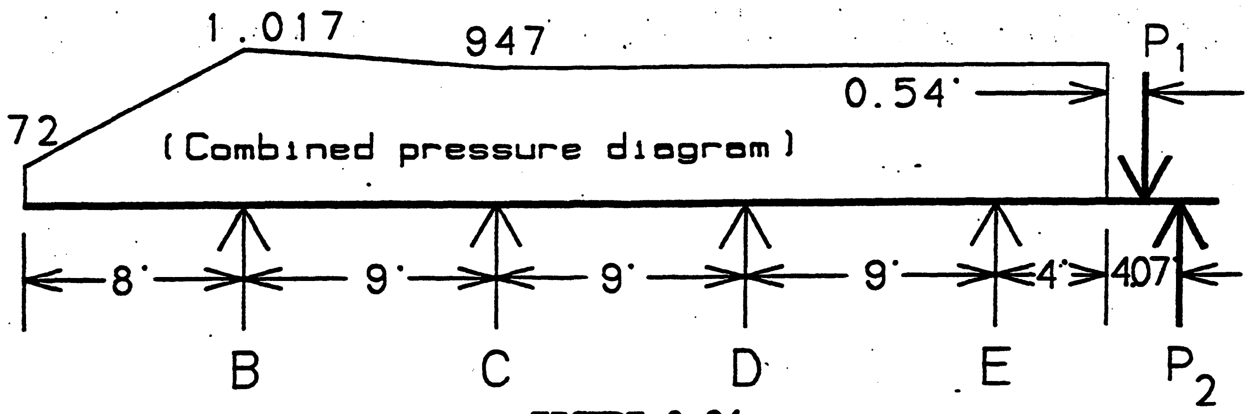


FIGURE 9-24

BC, CB, CD, DC, DE, ED = 4EI/L = 4/9 =0.444
(EI = constant)

Distribution Factors:

0.444/(0.444 + 0.444) = 0.5

BA = 0	CB = 0.5	DC = 0.5	ED = 1
BC = 1	CD = 0.5	DE = 0.5	EP_2 = 0

Rectangular Loads: $M = wL^2/12$

Triangular Loads: Light end, $M = wL^2/30$
Heavy end, $M = wL^2/20$

M_{BA} = (72)(8)[8/2] + {(1,017 - 72)(8)/2}[8/3] = 12,384

M_{BC} = (875 + 72)(9)²/12 + (142 - 72)(9)²/20 = 6,676

M_{CB} = (875 + 72)(9)²/12 + (142 - 72)(9)²/30 = 6,581

M_{CD} = M_{DC} = M_{DE} = M_{ED} = (875 + 72)(9)²/12 = 6,392

M_{EP2} = (947)(4)[2] + 709[4.54] - 3,662[8.07] = -18,757

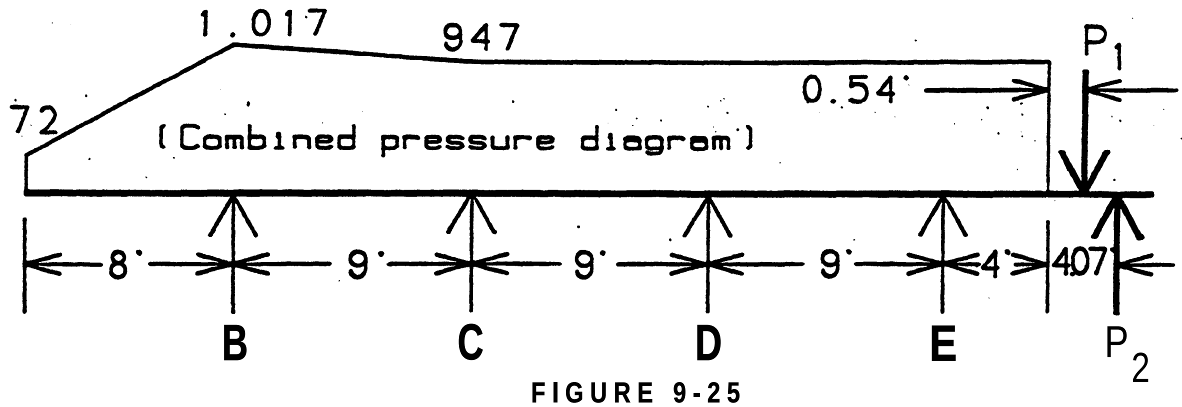


FIGURE 9-25

	0	1	0.5	0.5	0.5	0.5	1	0
	-12384	6676	-6581	6392	-6392	6392	-6392	-18757
		5708	95	94	0	0	25149	
		47	2854	0	47	12575	0	
		-47	-1427	-1427	-6311	-6311	0	
		-714	-24	-3156	-714	0	-3156	
		714	1590	1590	357	357	3156	
		795	357	178	795	1578	178	
		-795	-268	-268	-1187	-1187	-178	
		-134	-398	-593	-134	-89	-593	
		134	496	496	112	111	593	
		248	67	56	248	296	56	
		-248	-62	-61	-272	-272	-56	
		-31	-124	-136	-30	-28	-136	
		31	130	130	29	29	136	
		65	15	16	65	68	14	
		-65	-16	-15	-66	-66	-14	
M_{TOT}	-12384	12384	-3296	3296	-13453	13453	18757	-18757
V_M		1010	-1010	-1096	1096	3579	-3579	
V_{SB}	4356	4472	4367	4262	4262	4262	4262	835
V_{TOT}	4356	5482	3357	3166	5358	7841	683	835

Check Sheeting:

The maximum moment is M_{EP2} = 18,757 Ft-Lb/LF
S Required = M/f = (18,757) (12)/25,000 = 9.00 in^3

Analysis by the free earth support method permits the use of Rowe's Theory of Moment Reduction.

$p = (H + D)^4/EI = ((39 + 5.3)(12)\}^4/(30 \times 10^6)(20.6) = 129.2$
From-Rowe's moment reduction curves (see USS Steel Sheet

Piling Design Manual, page 32):

RATIO M_{DESIGN}/M_{MAX} = 0.34

$\therefore M_{DESIGN}$ = (0.34)(18,757) = 6,377 Ft-Lb/LF

S Required = (6,377)(12)/25,000 = 3.06 < 6.98 $\therefore$ O.K.

Check Wales:

$M \approx wL^2/10 \approx (13,199)(7.5)^2/10 \approx 74,244$ Ft-Lb

S Req'd = (74,244(12)/22,000 = 40.5 in^3

S furnished (W16 x 26) = 38.4 < 40.5 $\therefore$ Not satisfactory

Recommend using W14 x 30; S = 42.0

Determine Tieback Loads:

B: (T_1) = 7.5(9,838) = 73,785 Lb
C: (T_2) = 7.5(6,523) = 48,923 Lb
D: (T_3) = 7.5(13,199) = 98,993 Lb
E: (T_4) = 7.5(1,518) = 11,385 Lb

Tieback 1 (installed in 8" diameter drilled hole)

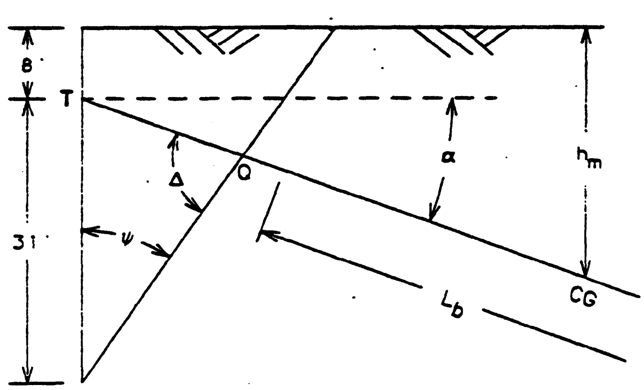


FIGURE 9-26

T_1Q = Unbonded length

L_b = Bonded Length

T_1 = 73,785 Lb

P = 73,785/cos 20°

P = 78,520 LB

ψ = 35°

$\Delta = 180° - (90-\alpha) - \psi = 75°$

$T_1Q = 31(\sin\psi)/\sin\Delta = 31(0.5736)/0.9659 = 18.4' > 15'$

h_m = Distance from ground surface to center of length L_b.

$$h_m = 8 + (T_1Q + L_b/2)\sin\alpha$$
$$h_m = 8 + (18.4 + L_b/2)(0.342) = 14.29 + 0.171L_b$$

Proof load = $P_{ULT} = P_{DESIGN}/0.8 = 98{,}150$ Lb

Use the FHWA formula to verify L_b

$$8{,}150 = \pi d\gamma h_m(\tan\phi)L_b$$
$$98{,}150 = \pi(0.667)(115)(14.29 + 0.171L_b)(\tan 35°)L_b$$
$$98{,}150 = 2{,}411.6L_b + 28.85L_b^2$$

$$L_b^2 + 83.58\,L_b - 3{,}402.08 = 0 \qquad L_b = 30'$$

And similar computations can be made for the other ties.

<u>Check Excavation Levels at 2' -0" Below Tie to be Installed:</u>

<u>EXCAVATE 10'</u>

EXCAVATION 19'

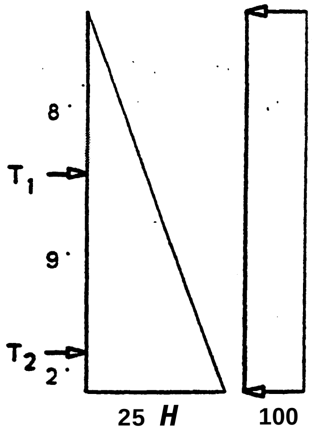


FIGURE 9-28

Before stressing T_2:

$$M_{19'} = [25(19)^3/6 + 100(19)^2/2] - 9{,}838(11)$$

$$= -78{,}689 \text{ Ft-Lb/LF}$$

Indicating a larger sheet pile section is needed.

The remainder of the ties need to be checked. A similar analysis might be required for backfill operations because the elevations used for stopping for installation of ties may not the same as that used for removal of ties.

Check Sheet Pile Penetration:

Use skin friction on the sides of the piling in contact with the soil; and-use skin friction working load value = 50% of ultimate value.

Skin friction = N/100 = Ultimate value

N from Table 12 = 20

N/100 = 20/100 = 0.2 tsf = 400 psf

Working load value = 50% of the ultimate, ∴ use 400/2 = 200 psf

Downward load from the T forces = $\sum T(\tan 20°)$ = 31,078 (0.364)

= 11,312 Lb/LF

11,312(7.5) = (39 + 2D)(7.5)(200) from which D = 8.8'

Use Safety Factor = 20% (Shape of the sheet piling was neglected)

Minimum D = 1.20(8.8) = 10.6 feet > 5.6 feet shown on plan.

SUMMARY

Multiple tieback systems approximate a multiple strutted system. The soil pressure diagram for either system should more appropriately approximate a trapezoid rather than a triangle. This would be especially true for soft to medium clays.

A long bond length is required at the elevation of the upper tier primarily because of the low h_m value. The tiebacks of the upper tier would have been better designed by reducing the center to center tie spacing to achieve a shorter required bond length. Another way to reduce the bonded length would be to locate the upper ties tiff-center with respect to the second tier ties and to increase the tie slope angle in order to increase the h_m value. The most practical way to decrease the length requirement of the upper tier tie would be to increase the diameter of the drilled hole to 16^n or to 18". This would. substantially increase 'the effective bond per linear foot of tie.

Three tiers of tiebacks properly spaced should have been adequate for the soil conditions and design parameters used in this case.